View metadata, citation and similar papers at core.ac.uk
brought to you by CORE
provided by Research Online

University of Wollongong
Research Online

Faculty of Engineering - Papers (Archive)
Faculty of Engineering and Information Sciences

1-1-2008

Ground improvement with special reference to non-toxic chemical stabilisation and deep sub-surface drainage

Buddhima Indraratna
University of Wollongong, indra@uow.edu.au

Jayan Sylaja J S Vinod
University of Wollongong, vinod@uow.edu.au

Cholachat Rujikiatkamjorn
University of Wollongong, cholacha@uow.edu.au

Follow this and additional works at: https://ro.uow.edu.au/engpapers

Part of the Engineering Commons

https://ro.uow.edu.au/engpapers/843

Recommended Citation

Indraratna, Buddhima; Vinod, Jayan Sylaja J S; and Rujikiatkamjorn, Cholachat: Ground improvement with special reference to non-toxic chemical stabilisation and deep sub-surface drainage 2008, 167-188.
https://ro.uow.edu.au/engpapers/843

Research Online is the open access institutional repository for the University of Wollongong. For further information contact the UOW Library: research-pubs@uow.edu.au

GROUND IMPROVEMENT WITH SPECIAL REFERENCE TO NON-TOXIC CHEMICAL STABILIZATION AND DEEP SUB-SURFACE DRAINAGE

Buddhima Indraratna[1], Jayan Sylaja Vinod[2] and Cholachat Rujikiatkamjorn[3]

[1]Professor of Civil Engineering, Faculty of Civil Engineering, University of Wollongong, NSW 2522, Australia; *indra@uow.edu.au*

[2]Research Fellow, Faculty of Civil Engineering, University of Wollongong, NSW 2522, Australia; *vinod@uow.edu.au*

[3]Lecturer, Faculty of Civil Engineering, University of Wollongong, NSW 2522, Australia; cholachat@uow.edu.au

ABSTRACT

In this paper, two advanced ground improvement schemes have been introduced to improve soft and erodible soil properties. The improvement of an unstable formation soil with pH neutral chemical admixture and the sub-surface drainage is described. Internal erosional behaviour of lignosulfonate treated erodible soils has been studied using the Process Simulation Apparatus for Internal Crack Erosion (PSAICE) designed and built at UoW. Effectiveness of lignosulfonate treated erodible soils on the erosion resistance has been investigated and its advantage over conventional methods are presented and discussed. For soft soil formation stabilised by sub-surface drainage, the paper will look at the inevitable problems of smear associated with mandrel driven Prefabricated Vertical Drains (PVDs), the distribution of suction pressures through PVD, and the conceptual development of consolidation models capturing the effects of vacuum. The applications to case histories employing equivalent plane strain theory developed by Indraratna and co-workers will be presented and discussed.

1. INTRODUCTION

Highly erodible and soft compressible soils are common in many parts of the world. In recent years, innovative soil improvement schemes have been developed, namely chemical stabilization and deep sub-surface drainage stabilization. Soft and unstable soils are conventionally stabilised using chemical admixtures using tradition stabilizers such as lime, gypsum, cement, fly ash etc. These admixtures (stabilizing agents) generally alter the mineralogical structure of the clay and improve the inherent properties of the soil such as strength and stiffness. Vast numbers of studies were conducted to investigate the applicability of traditional stabilisers on problematic soils such as soft clay and erodible soils (e.g. Indraratna *et al.* 1991; Uddin *et al.* 1997; Balasubramaniam *et al.* 1998; Indraratna *et al.* 1995; Rajasekaran *et al.* 1997, Chew *et al.* 2004).

Problems such as sulphate attack on concrete and steel structures adjacent to gypsum treated soils, problems with vegetation and groundwater contamination of chemically treated soils due to high pH levels etc, have demanded researchers to find alternative stabilizers. Recently, lignosulfonate, a by product of wood industry shows a promising prospect as a stabilizing agent especially for soft ground. Preliminary investigation on lignosulfonate as stabilizing agent showed that, amount of lignosulfonate required to stabilize soft/unstable soils is much less compared to other admixtures such as cement, lime, or fly ash. Furthermore, it has also been observed that stress strain and volume change behaviour is distinctly different from those stabilised with traditional admixtures and that the lignosulfonate soils maintain the ductile characteristics. Only a few studies are reported in the literature, hence, the mechanism of soft soil stabilization by lignosulfonate is not well understood.

For very thick soft soil deposits, the combination of part surcharge and vacuum pressure with prefabricated vertical drains (PVD) has become an attractive alternative. The vertical drains can be used to accelerate the consolidation process thereby increasing the undrained shear strength of the soil. *Centre for Geomechanics and Railway Engineering* at University of Wollongong is one of the leading soft soil testing and ground improvement centres in the world, and through its various programmes, the use of large scale experimental rigs, theoretical development and numerical simulations of soft clay improvement with vacuum preloading via vertical drains have contributed to significant advancement of the design techniques during the past decade.

In this paper, laboratory experiments were carried out on erodible soils stabilised with lignosulfonate, and the current study focuses on the effectiveness of lignosulfonate treatment on the erosional behaviour of different soils (a silty sand and a dispersive clay). Laboratory experiments were carried out on a process simulation apparatus for internal crack erosion that was designed and built at the University of Wollongong.

In terms of soft formation stabilised by sub-surface drainage, the paper will look at the inevitable problems of smear associated with mandrel driven PVDs, the distribution of suction pressures through PVD, conceptual development of consolidation models capturing the effects of vacuum. The applications to case histories employing equivalent plane strain theory will be presented and discussed.

2. SOIL STABILIZATION USING LIGNOSULFONATE

Lignosulfonates were commonly used to stabilise cohesive to non-cohesive soils. These stabilisers are made from waste liquor by-products from wood processing industries such as paper mills (Karol, 2003). For stabilization purposes, solutions of lignosulfonate were used as raw liquor or used with other additives to achieve desire soil properties. In the recent past, investigations were carried out on cohesive soils with lignosulfonate as stabilisers on the strength improvement of cohesive soils (Puppala and Hanchanloet, 1999; Pengelly *et al.* 1997; Tingle and Santori, 2003). It has been reported that lignosulfonate with sulphuric acid as additive showed a profound increase in their shear strength and resilient modulus. Tingle and Santori (2003) investigated the effect of lignosulfonate on different clayey soils and found that lignosulfonate stabiliser significantly improved the strength of a low plasticity clayey soil. Again, a solution containing ammonium lignosulfonate and potassium chloride was injected into expansive soil and a significant reduction in the swelling was observed (Pengelly *et al.* 1997). In addition, a number of researchers have performed experiments to investigate whether this particular type of chemical in low volume road construction would improve the strength of sub-grade and control dust emission (e.g. Chemstab 2003; Tingle and Santori 2003; Lohnes and Coree 2002).

Studies have been carried out by various researchers in the recent past to understand the erosion mechanism and its dependability on different factors such as soil properties, and the properties of pore and eroding fluids. Sherard *et al.* (1976) developed the standard pinhole test to study the erosion characteristics of soil by pushing eroding fluid through a 1-mm crack. Wan and Fell (2004) performed erosion tests by applying a hydraulic gradient across a 6 mm internal crack to study the erosion characteristics of unsaturated soil in cracks of embankment dams. In the present study, internal crack erosion behaviour of chemically treated soil has been investigated using Process Simulation Apparatus for Internal Crack Erosion (PSAICE). All the tests were carried out by pushing the eroding fluid through a 10 mm soil crack formed at the centre of the sample.

2.1 Materials and Methods

A series of internal crack erosion tests were conducted on two natural erodible soils, a dispersive clayey soil and a silty sand collected from different parts of New South Wales (NSW), Australia. According to the standard pinhole test (ASTM D 4647), the silty sand and the dispersive clay are classified as D1 and D2, respectively. Particle size distribution of these soils is shown in Fig. 1. The maximum dry density and optimum moisture content for silty sand was determined to be 17 kN/m^3 and 10.3% respectively. However, for dispersive clay these properties were observed to be 15 kN/m^3 and 22% respectively. Furthermore, the liquid limit and plastic limit of the dispersive clay were found to be 47.6 % and 29.4 % respectively. Internal crack erosion studies were conducted using the Process Simulation Apparatus for Internal Crack Erosion (PSAICE), which has developed and built at University of Wollongong.

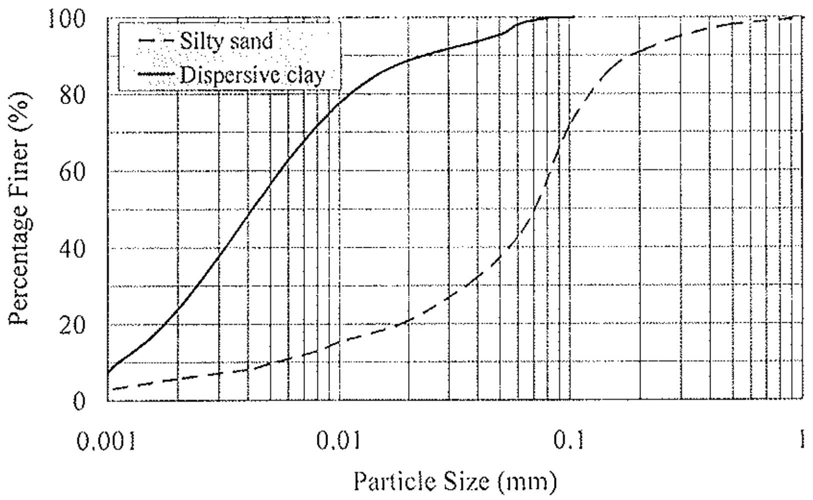

Fig. 1 Particle size distribution curve

Figure 2 shows the photograph of the test apparatus (PSAICE). This equipment has an adjustable head tank capable of applying a hydraulic gradient of up to 40 across the samples. Pressures across the samples were measured using electronic transducers and an inline turbidity meter was installed downstream of the sample to continuously monitor the effluent turbidity. The effluent was continuously weighed with an electronic balance in order to measure the flow rate. Detailed explanations of the testing equipment can be found elsewhere (e.g. Indraratna et al., 2008; and Thevaragavan, 2008).

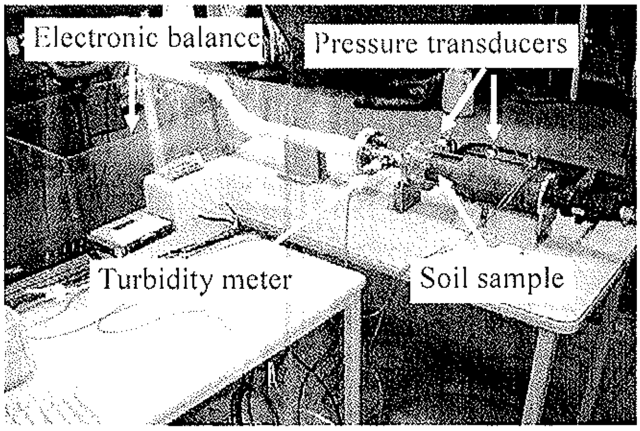

Fig. 2 Photograph of Process Simulation Apparatus for Internal Crack Erosion (PSAICE)

Various amounts of lignosulfonate [(0.1% - 0.6%), by dry weight of soil] were selected to stabilise the erodible soils. Both soils (silty sand and dispersive clay) were mixed with the selected amount of lignosulfonate additive and statically compacted to 95 % of the dry density inside a copper mould having a size of 72 mm diameter and 100 mm height. The prepared samples were wrapped in moisture proof bag and cured for seven days. After curing, these samples were immersed in the eroding fluid (tap water) until saturation. Internal crack erosion tests were then carried out by pushing the eroding fluid through a 10-mm soil crack formed at the centre of the samples using the PSAICE.

2.2. Theoretical Background

Internal erosion behaviour of lignosulfonate treated and untreated soils were studied using PSAICE. Predicted erosion rate and hydraulic shear stress were used to calculate the erosion parameters, namely, the critical shear stress and the coefficient of soil erosion. The critical shear stress, τ_c, is defined as the minimum hydraulic shear stress necessary to initiate erosion. Figure 3 shows the variation of the erosion rate with the hydraulic shear stress. The critical shear stress is estimated by extrapolating the straight line to the zero erosion rate. The slope of the linear line was presumed to be the coefficient of soil erosion.

Figure 4 shows the typical plot of effluent turbidity and flow rate with time. It is evident from the Fig. 4 that the turbidity increases initially, and then decreases as erosion progresses. However, the flow rate was observed to be increasing with time. The amount of soil eroded in a particular time interval δt can be represented as:

$$\delta m = kQT \times \delta t \qquad (1)$$

where, δm (kg) is the amount of dry soil eroded during a selected time interval δt, Q (m^3/s) is the average flow rate through the soil crack at time interval δt; T (NTU) is the average turbidity of the effluent at δt; and k (kg/m^3/NTU) is the empirical factor relating turbidity to the soil solids concentrated in the flow.

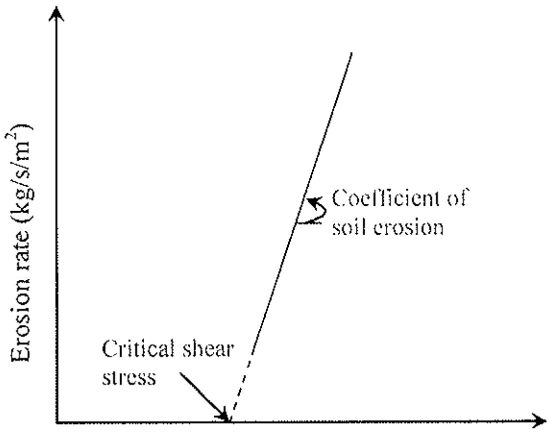

Fig. 3 Typical plot of erosion rate versus hydraulic shear stress

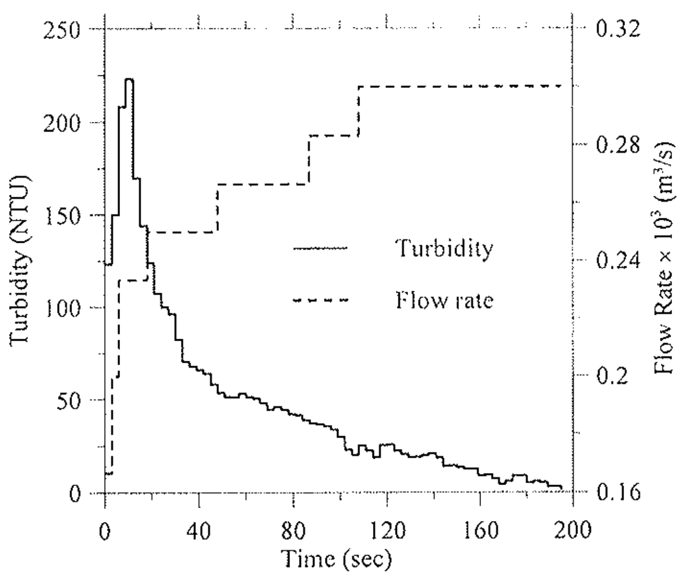

Fig. 4 Typical plot of turbidity and flow rate with time for untreated dispersive clay

The value of k was evaluated from the linear relationship between soil concentration and turbidity. Figure 5 presents a typical plot of soil concentration with turbidity. The value was estimated to be 0.011 kg/m^3/NTU for lignosulfonate treated silty sand and 0.002-0.011 kg/m^3/NTU was observed for treated and untreated dispersive clay. In addition, when the diameter of the soil crack changes by $\delta\phi_t$ in a time interval δt, the amount of soil eroded during this time will be:

$$\delta m = \frac{\pi \phi_t l \rho_d}{2} \times \delta \phi_t \tag{2}$$

Where, ρ_d (kg/m^3) is the dry density of compacted soil; l (m) is the length of the soil crack; and ϕ_t (m) is the diameter of the soil crack at time t.

Equating expressions (1) & (2)

$$\delta \phi_t = \frac{2kQT}{\pi \phi_t l \rho_d} \times \delta t \tag{3}$$

The erosion rate, $\dot{\varepsilon}$ (kg/s/m^2), can then be calculated using Equation (4)

$$\dot{\varepsilon} = \frac{kQT}{\pi \phi_t l} \tag{4}$$

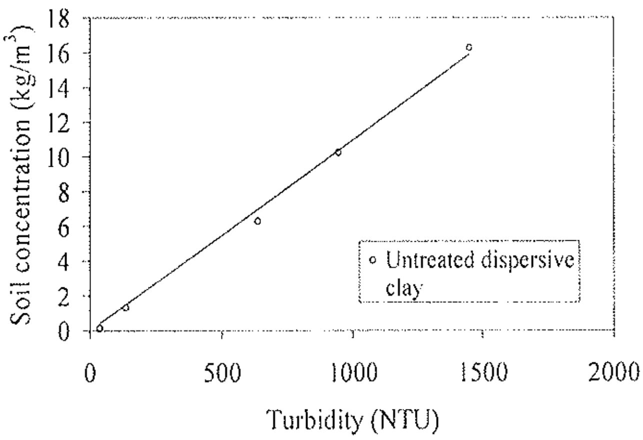

Fig. 5 Variation of soil concentration with turbidity for untreated dispersive clay

2.2.1 *Estimation of Hydraulic Shear Stress from Friction Factor Method*

The hydraulic shear stress is estimated from:

$$\tau_a = \frac{f \rho_w v^2}{8} \tag{5}$$

where, f is the friction factor, ρ_w (kg/m^3) is the density of the eroding fluid; and v (m/s) is the mean velocity of the flow through the crack at time t, which can be calculated using the flow rate and diameter of the crack. The friction factor was calculated from the Moody diagram (Abulnaga, 2002) based on the relative roughness and Reynolds number. The relative roughness is calculated from

$$\varepsilon = \frac{D}{2\phi_t} \tag{6}$$

where, D (m) is the mean particle diameter. The height of the roughness element was taken as the radius of the mean particle. The mean particle diameter was estimated from particle size

distribution of the eroded particles obtained from the Malvern Mastersizer. The change in mean particle size for specimens of silty sand (untreated and treated) was negligible, but it changed for dispersive clayey specimens (untreated and treated). For dispersive clay, the change in diameter was observed to be 19, 22 and 25 microns for 0.2%, 0.4% & 0.6% lignosulfonate respectively for samples compacted at 95% of the maximum dry density.
The Reynolds number can be calculated using Equation (7):

$$R_e = \frac{\rho_w v \phi_i}{\mu} \quad (7)$$

Where, μ ($kgm^{-1}s^{-1}$) is the dynamic viscosity of the eroding fluid

2.3 Results And Discussions

Figure 6 presents the variation of erosion rate with hydraulic shear stress for lignosulfonate treated and untreated silty sand at 95% of the maximum dry density. It is evident from the Fig. 6 that the erosion rate and hydraulic shear stress follow a linear relationship and the slope represents the coefficient of soil erosion. As expected, critical shear stress increases and coefficient of soil erosion decreases with the increase in the amount of lignosulfonate. When the amount of lignosulfonate is increased to 0.6 % the critical shear stress increases from 0.8 Pa to 25 Pa. In addition, the coefficient of soil erosion decreases from 0.265sm^{-1} to 0.003 sm^{-1}. Lignosulfonate treated dispersive clay also exhibited a similar behaviour (critical shear stress increases and coefficient of soil erosion decreases, with the amount of lignosulfonate) as that of silty sand (Fig. 7). The critical shear stress increases from 3.6 Pa to 27 Pa with the addition of 0.6 % of lignosulfonate and coefficient of soil erosion decreases from 0.019 sm^{-1} to 0.0012 sm^{-1}.

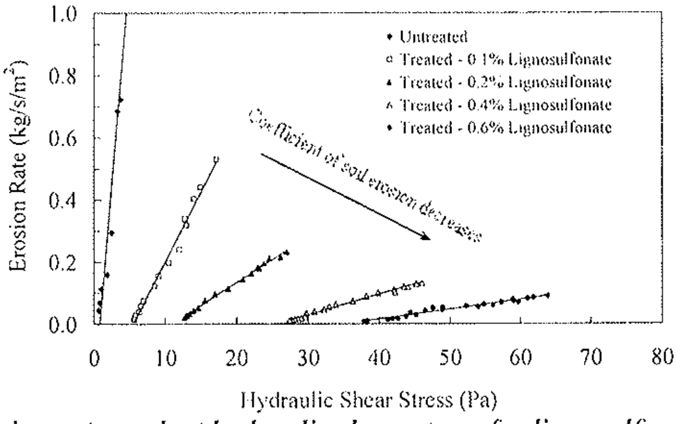

Fig. 6 Erosion rate against hydraulic shear stress for lignosulfonate treated and untreated silty sand

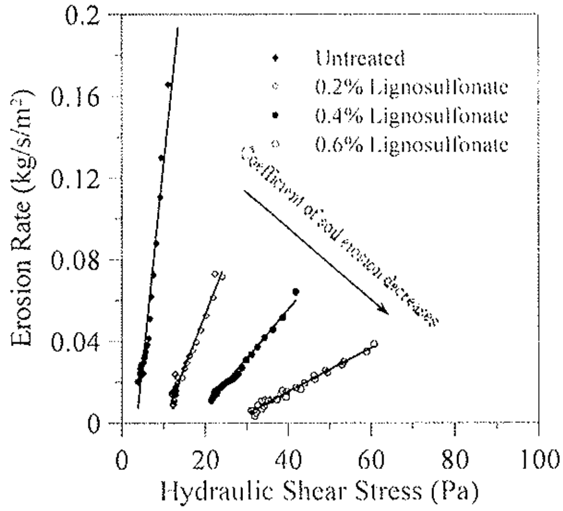

Fig. 7 Erosion rate against hydraulic shear stress for lignosulfonate treated and untreated dispersive clay

2.3.1 Comparison with Traditional Admixtures

The behaviour of lignosulfonate treated soils (silty sand and dispersive clay) has been compared with cement treated soils. General purpose Portland cement was used for soil stabilisation and erosion tests were carried out on cement treated soils very similar to lignosulfonate stabilised soils. Figures 8 and 9 present the variation of erosion rate versus hydraulic shear stress for silty sand and dispersive clay, respectively. It can be observed from Figures 8 and 9 that critical shear stress increases and the coefficient of soil erosion decrease with the increase in cement. This behaviour is very similar to that reported for lignosulfonate treated soils (Figures 6 and 7).

The performance improvement in terms of critical shear stress due to addition of chemical additives (lignosulfonate and cement) can be represented as a non-dimensional Critical Shear Stress Ratio (CSSR), which is defined as the ratio of critical shear stress of treated soil to the critical shear stress of untreated soil.

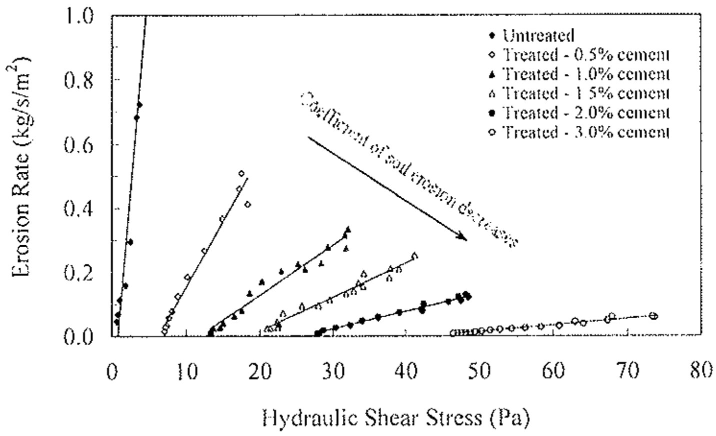

Fig. 8 Erosion rate against hydraulic shear stress for cement treated and untreated silty sand

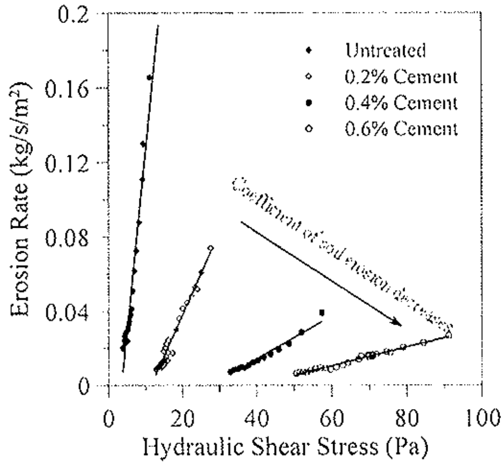

Fig. 9 Erosion rate against hydraulic shear stress for cement treated and untreated dispersive clay

The variation of critical shear stress ratio for lignosulfonate and cement treated soils (silty sand & dispersive clay) is tabulated in Table 1. It is evident from Table 1 that for silty sand CSSR of 7.5 was achieved with 0.1 % of lignosulfonate. In contrast, for cement treated silty sand the same value of CSSR (7.5) was observed at 0.5 %. For silty sand the performance improvement in terms of CSSR was observed to be higher for a lesser amount of lignosulfonate when compared to cement. However, for dispersive clay, the performance improvement in terms of CSSR was observed to be slightly higher for cement treated soil when compared to lignosulfonate. Studies are in progress to understand the stabilization mechanism involved in the lignosulfonate treated dispersive clay.

Table 1 Variation of critical shear stress ratio for different lignosulfonate and cement treated soils

Soil Type	Amount of chemical (%)	Critical Shear Stress Ratio (CSSR)	
		Lignosulfonate	Cement
Silty sand	0.1	7.5	-
	0.2	13.8	-
	0.4	31.3	-
	0.5	-	7.5
	0.6	43.8	-
	1	-	16.3
	1.5	-	23.8
	2	-	31.3
	3	-	53.8
Dispersive clay	0.2	2.5	2.7
	0.4	4.5	6.5
	0.6	6.7	9.8

Digital images using Scanning Electron Microscope (SEM) were used to understand the stabilization mechanism of lignosulfonate treated silty sand. As can be seen in Fig. 10, untreated soil grains are distinctly separate with clear boundaries between them. However, with lignosulfonate addition (Fig. 11), the particles become coated with this chemical, which bonds them closely together to produce a stronger soil structure. Based on these observations from the SEM analysis, it may be concluded that the lignosulfonate stabiliser act like cementing agents to bind the particles together to form an erosion resistant surface

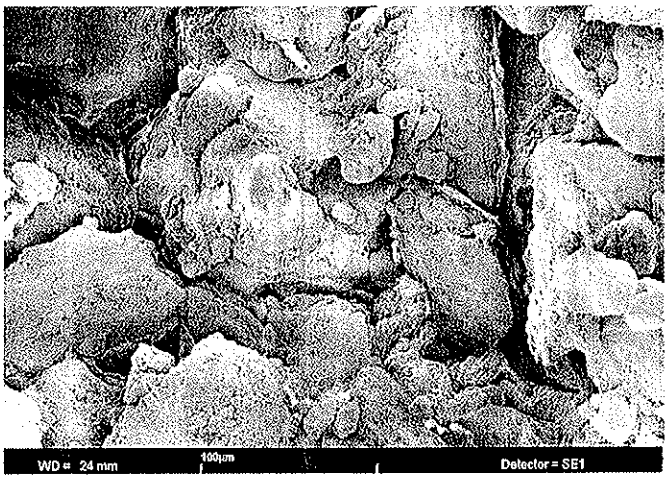

Fig.10 SEM photograph-untreated silty sand (after Indraratna et al. 2008)

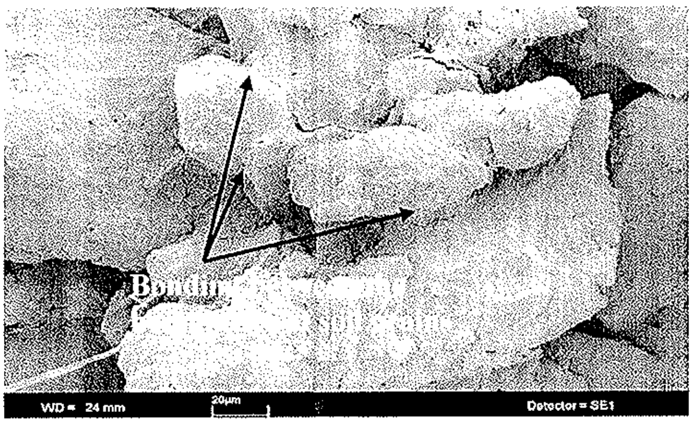

Fig.11 SEM photograph - 0.2% lignosulfonate treated silty sand (after Indraratna et al. 2008)

3. GROUND IMPROVEMENT USING DEEP SUB-SURFACE DRAINAGE

3.1 Purpose and Application of Vertical Drains

The vertical drains can be used to accelerate the consolidation process as they shorten drainage path. Vertical drains allow the higher horizontal permeability to be exploited. There are two methods to install vertical drains: displacement and non-displacement (Bo et al. 2003). The non-displacement installation engages pre-boring of holes which later are filled up by more permeable material; usually sand Displacement type installation is conducted by forcing the vertical drains into the soil with a hollow mandrel using static or dynamic force. Prefabricated vertical drains (PVD) consisting of a core surrounded by a filter sleeve are commonly used in the field due to their fast installation and less soil disturbance.

3.2 Application of Vertical Drains and Vacuum Pressure

When the shear strength of soil is very low (e.g. reclamation land), the application of surcharge loading alone may be too slow or inappropriate for the site. In such cases vacuum preloading technique can be used in conjunction with PVDs (Shang et al. 1998; Yan and Chu, 2003; Indraratna et al., 2005). Application of vacuum load can further accelerate the rate of settlement under constant total stress (Qian et al., 1992). An external suction is applied via vacuum pump to the soil surface in the form of vacuum through a sealed membrane system.
Currently, there are two types of vacuum preloading system. Their effectiveness depends on the soil type and the nature of drains installed.

3.2.1 Vacuum preloading system with membrane

A permeable drainage layer (sand blanket) is placed to cover the horizontal drains network after PVDs installation. The membrane is then laid over the sand blanket to ensure an airtight region above the improved area (Fig. 12). The edge of the membrane is submerged in a peripheral trench filled with bentonite slurry to prevent any air leaks. The vacuum pumps are then connected to the discharge system extending from the drain network. A benefit of this system is that the negative pressure propagates along the soil surface and down the PVDs, accelerating the dissipation of excess pore water pressure both horizontally towards the PVDs and vertically towards the sand blanket. However, the efficiency of the entire system depends on the ability to prevent air leaks in order to sustain sufficient suction over a significant period of time (Indraratna *et al.* 2004).

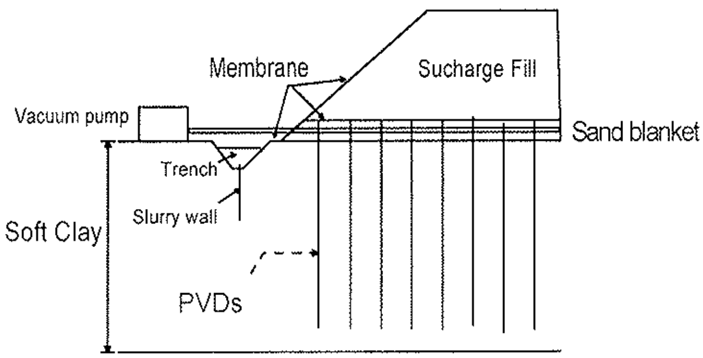

Fig. 12. Schematic diagram of membrane system with vacuum preloading

3.2.2 Membraneless vacuum preloading system

In this approach, the vacuum channel is directly connected to each individual PVD using a tubing system, which connects directly to the drain collector (Fig. 13). In contrast to the membrane system where any air leak can affect overall the system efficiency, each drain acts independently. Nevertheless, the requirement for extensive tubing connection for hundreds of drains will significantly increase the installation time and cost (Seah, 2006).

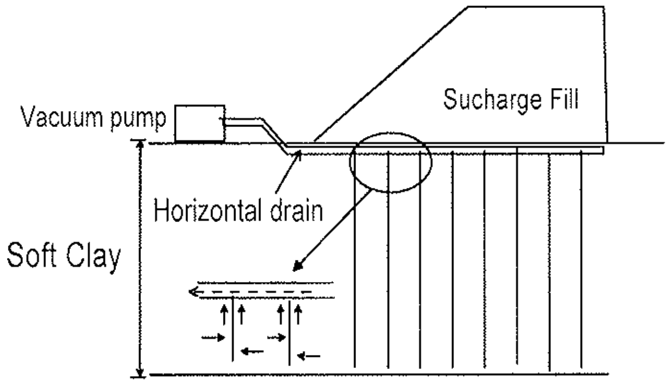

Fig. 13 Schematic diagram of membraneless vacuum preloading system

3.3 Design Considerations for Vertical Drain System

3.3.1 Discharge Capacity

The discharge capacity (drain permeability) is one of the most important parameter that controls the rate of consolidation. The discharge capacity depends on the following factors (Fig. 14): (a) the drain cross section; (b) the effect of confining pressure; (c) folding, bending and kinking of the drain in the soft zone and (d) efficiency of drain infiltration to prevent clogging.

Holtz et al. (1991) recommended that as long as the working discharge capacity of PVD exceeds 150 m^3/year, the effect on consolidation should not be significant. Indraratna and Redana (2000) confirmed that long term well resistance will be significant for PVD with q_w less than 40-60 m^3/year. However, discharge capacity can be lower than the minimum value due to the reasons aforementioned. The discharge capacity may reduce to 25-100 m^3/year by significant vertical compression and high lateral pressure (Holtz et al., 1991). Clearly, the 'clogged' drains can be described as q_w values approach zero.

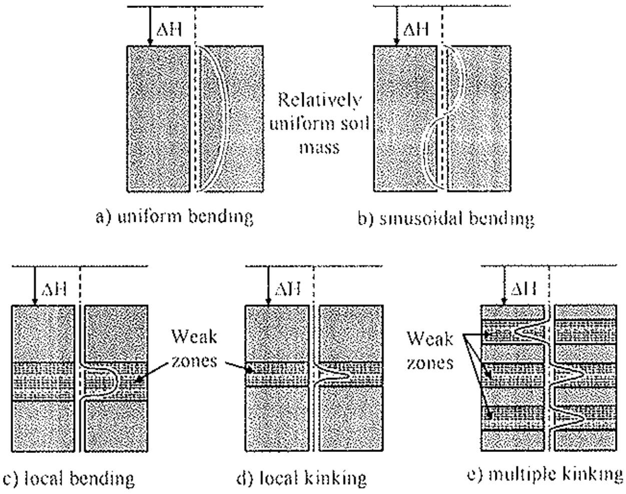

Fig. 14 Modes of PVD Deformation (after Bergado et al., 1996)

3.3.2 *Drain unsaturation*

Unsaturation of soil adjacent to the drain can occur due to mandrel withdrawal (air gap) during the PVDs installation. Indraratna et al. (2004) attempted to describe the apparent retardation of pore pressure dissipation in large-scale laboratory testing through a series of models, considering the effects of unsaturation at the drain-soil interface.

3.3.3 *Smear zone*

The smear zone is created during a PVD is installed using displacement installation technique. Soil permeability adjacent to PVDs can reduce substantially, which in turn retards the excess pore pressure dissipation and the consolidation process. The reduction in soil permeability can be determined either by the ratio of average soil permeability in the smear zone and the in-situ horizontal soil permeability (k_h/k_s) or the variation of water content (Indraratna and Redana, 1998; Indraratna and Sathananthan, 2006). This k_h/k_s ratio and the size of smear zone (d_s) depend on the soil sensitivity, mandrel shape, installation speed, and the soil macro fabric. It is observed that the smear zone diameter is about 100 mm or 1.6 times the mandrel diameter and the ratio of k_h/k_s is approximately 2. Walker and Indraratna (2007) showed that the smear effects can overlap creating more soil disturbance when vertical drains are installed too close (Fig. 15).

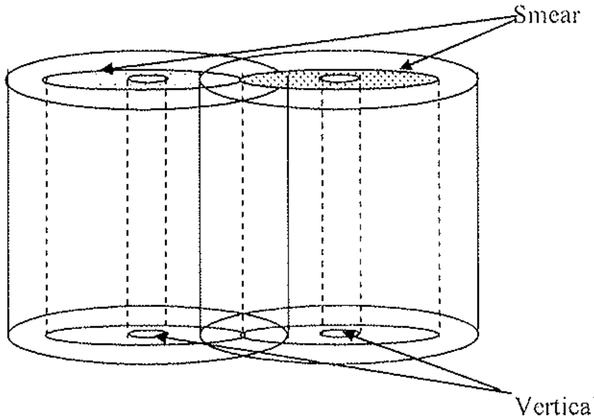

Fig. 15 Overlapping smear zones between adjacent drains (after Walker and Indraratna, 2007)

3.4 Calculation of the Degree of Consolidation

The degree of consolidation can be determined based on excess pore pressure or settlement. The degree of consolidation based on the settlement can be calculated as:

$$U = \frac{s_t}{s_{ult}} \tag{8}$$

where, s_t = settlement measured at time t, s_{ult} = ultimate settlement
The degree of consolidation based on excess pore pressure can be expressed as (Fig 16):

$$U_p = \frac{h_w\gamma_w + h_{fill}\gamma_{fill} - u_t}{h_{fill}\gamma_{fill} + |VP|} = \frac{\Delta u_t}{\Delta\sigma + |VP|} \tag{9}$$

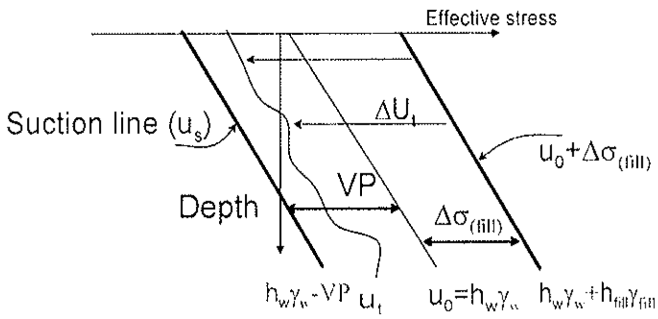

Fig. 16 Pore pressure distribution with depth

3.5 Consolidation Theory Considering Radial Drainage

The first solution for predicting radial consolidation was proposed by Barron (1948). For equal strain conditions, there is no differential settlement in horizontal sections. Barron also included the effects of smear assuming the constant reduced permeability. The schematic diagram of a soil cylinder with a central vertical drain is shown in Fig. 17, where, d_w = equivalent drain diameter, d_s = smear zone diameters, d_e = soil cylinder diameter and l = the length of the drain installed into the soft ground. The permeability in the vertical and horizontal directions are k_v and k_h. k_s is the permeability in the smear zone. The degree of consolidation can be expressed by:

$$U_h = 1 - \exp\left(\frac{-8T_h}{\mu}\right) \tag{10}$$

where,

$$T_h = \frac{c_h t}{4r_e^2} \tag{11}$$

If the constant smear zone is assume, the μ parameter is:

$$\mu_C = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right)\ln(s) - 0.75 \tag{12}$$

For the linear variation of permeability in the smear zone, the μ parameter can be (Walker and Indraratna, 2007):

$$\mu_L = \ln\left(\frac{n}{s_L}\right) - \frac{3}{4} + \frac{\kappa_L(s_L - 1)}{s_L - \kappa_L}\ln\left(\frac{s_L}{\kappa_L}\right) \tag{13}$$

For the case when $s_L = \kappa_L$ the μ parameter is:

$$\mu_L = \ln\left(\frac{n}{s_L}\right) - 1.75 + s_L \tag{14}$$

If the smear zones are overlapping, μ can be calculated based on:

$$\mu_X = \begin{cases} \mu_L[n, s_L, \kappa_L] & n \geq s_L \\ \frac{\kappa_L}{\kappa_{LX}}\mu_L[n, s_{LX}, \kappa_{LX}] & 2n - s_L \geq 1, \text{ and, } s_L > n \\ \frac{\kappa_L}{\kappa_{LX}}\mu_I[n] & 2n - s_L < 1 \end{cases} \tag{15}$$

$$\kappa_{LX} = 1 + \frac{\kappa_L - 1}{s_L - 1}(s_{LX} - 1) \tag{16}$$

$$s_{LX} = 2n - s_L \tag{17}$$

$$\mu_I = \ln(n) - 0.75 \tag{18}$$

In the above equations $n = d_e/d_w$ is the drain spacing ratio and $s = d_s/d_w$ is the smear zone size ratio.

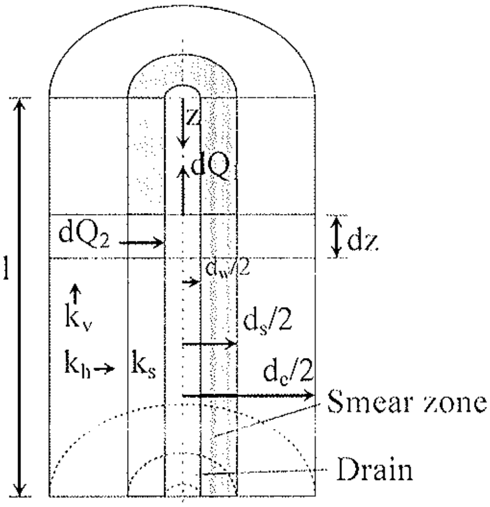

Fig. 17 Schematic of soil cylinder with vertical drain (after Hansbo, 1981)

3.6 Multi- drain Analysis for a Plane Strain Embankment

Indraratna et al. (2005) transformed the vertical drain system from axisymmetric (3D) to plane strain condition by determining the equivalent coefficient of soil permeability.
A relationship between k_{hp} and k'_{hp} can be modelled as:

$$\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\frac{k_{hp}}{k_h}\left[\ln\left(\frac{n}{s}\right)+\left(\frac{k_h}{k'_h}\right)\ln(s)-0.75\right]-\alpha} \tag{19}$$

$$\mu_p = \left[\alpha+(\beta)\frac{k_{hp}}{k'_{hp}}\right] \tag{20}$$

$$\alpha = \frac{2}{3} - \frac{2b_s}{B}\left(1-\frac{b_s}{B}+\frac{b_s^2}{3B^2}\right) \tag{21}$$

$$\beta = \frac{1}{B^2}(b_s-b_w)^2 + \frac{b_s}{3B^3}\left(3b_w^2-b_s^2\right) \tag{22}$$

where, k_{hp} and k'_{hp} are the undisturbed horizontal coefficient of permeability and the corresponding equivalent coefficient of permeability in smear zone, respectively.
If the smear and well resistance are ignored in the above expressions, the simplified ratio of plane strain to axisymmetric permeability can be obtained, as also proposed earlier by Hird *et al.* (1992), to be:

$$\frac{k_{hp}}{k_h} = \frac{0.67}{[\ln(n)-0.75]} \tag{23}$$

For vacuum preloading, the equivalent vacuum pressures under plane strain and axisymmetric conditions are the identical.

3.7 Application to a Case History

3.7.1 Site Descriptions

The Sunshine Motorway is located in Maroochy Shire, Queensland, Australia. Site investigation showed that the soil layers are highly compressible, saturated estuarine clays. Ground improvement scheme is necessary to stabilise the soil before the highway construction. In order to assess the effectiveness of ground improvement techniques, a fully instrumented trial embankment was constructed in 1992 and monitored by the Queensland Department of Main Roads (QDMR), Brisbane.

The trial embankment was divided into 3 subsections (Sections A, B, and C). Sections A and B represented the zones of PVDs installed at 1m intervals and 'no drains', respectively. PVDs at 2m spacing were installed in Section C. The embankment was constructed up to a height of 2.3m with two berms, 5m and 8m on each side of the embankment (Fig. 18). Half of the cross-section was intensively instrumented with settlement plates, piezometers and inclinometers to capture the foundation response upon loading.

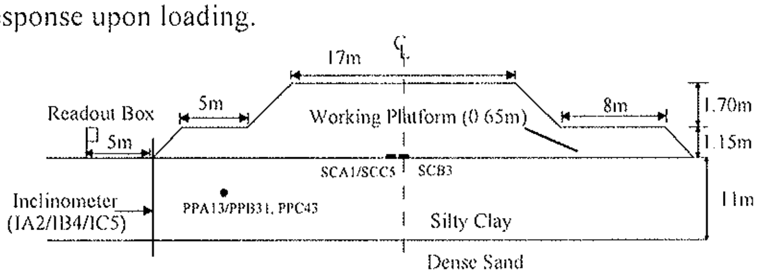

Fig. 18 Cross-section of embankment with selected instrumentation points (adapted from Rankine et al 2008)

3.7.2 Finite element analysis

Sathananthan et al. (2008) conducted the multi-drain plane strain analysis using the finite element code PLAXIS. The soft soil model based on Modified Cam-clay theory was employed to analyse the behaviour of clay layers. Based on the standard consolidation test results on vertical and horizontal specimens, the horizontal permeability (k_h) is 2 times the vertical permeability (k_v). For the embankment surcharge (silty sand), the Mohr-Coulomb model was used. The laboratory determined soil parameters are given in Table 2.

Table 2 Modified Cam-clay parameters used in the finite element analysis (adopted from Sathananthan et al. 2008)

Depth (m)	Soil type	M	$\lambda/(1+e_o)$	$\kappa/(1+e_o)$	ν	e_0	γ kN/m³
0.0-2.5	Silty clay	1.20	0.19	0.019	0.30	1.6	16.4
2.5-5.0	Soft silty clay	1.20	0.63	0.063	0.30	2.2	13.7
5.0-11	Silty clay	1.18	0.19	0.019	0.3	1.8	15.9

3.7.3 Results and Discussions

Figure 18 presents the predicted and measured surface settlements. The predictions agree well with the field measurement data for Sections B and C. It shows that the rate of settlement increases due to the installation of PVDs. The installation of closely spaced drains at Section A causes greater smear thereby marginally increasing the consolidation rate. This demonstrates that reducing the drain spacing excessively may only provide a marginal advantage due to overlapping smear.

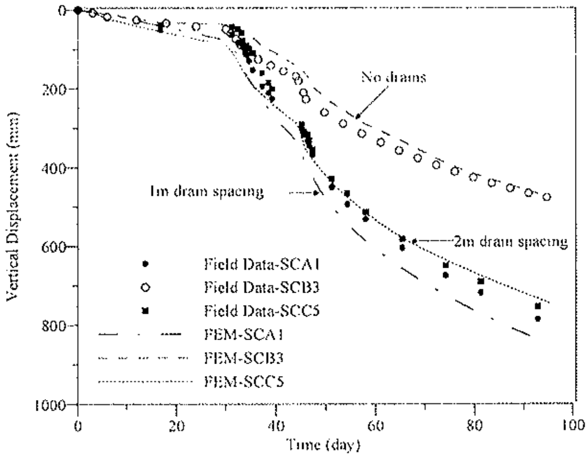

Fig. 19 Vertical displacement at the centerline (After Sathananthan et al., 2008)

The predicted and measured excess pore pressures beneath the middle of the berm are shown in Fig. 20. The excess pore pressures were built up due to embankment loading, and the predictions agree well with the field data. The excess pore pressure at Section B (no drains) is significantly higher (than the other sections approximately 30%. Surprisingly, at Section A where the drains are installed closely, the excess pore water pressure dissipation rate is almost the same as Section C. This can be attributed to excessive disturbance at Section A.

Lateral deformation at the edge of the 5m wide berm at Sections A, B and C are plotted with the numerical predictions in Fig. 21. As expected, PVDs significantly restrain the lateral displacement. For example, at 1m below the ground surface, the PVDs with 1m spacing (Section A) decreased the lateral displacement by one-fifth compared to Section B (no drains).

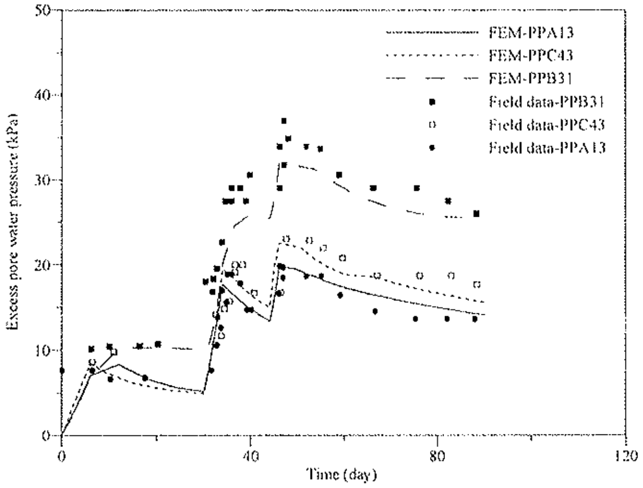

Fig. 20 Time-dependent excess pore pressure beneath the middle of the berm 5m (After Sathananthan et al., 2008)

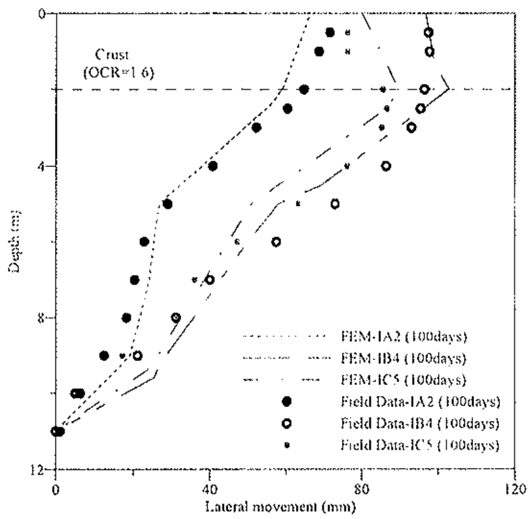

Fig. 21 Lateral displacement at Embankment toe (100 days after embankment construction)

CONCLUSIONS

In this paper, soil improvement methods including non-toxic chemical stabilization and deep sub-surface drainage were presented with their application to soft and erodible soil. For non-toxic chemical stabilization, the experimental method was conducted to evaluate the critical shear stress and the coefficient of soil erosion stabilised bylignosulfonate. Results of the present study show the potentiality of lignosulfonate to stabilise the erodible soils against internal crack erosion. It was found that the lignosulfonate stabilizer would reduce the coefficient of soil erosion and significantly increased the critical shear stress for both clayey and silty soils. The performance improvement of critical shear stress was expressed in terms of critical shear stress ratio (CSSR). The increase in the critical shear stress of the silty sand with only 0.6%

lignosulfonate treatment was equivalent to that with around 2.5% cement treatment. However, the stabilisation of dispersive clay was more effective with 0.6% cement than 0.6% of lignosulfonate.

Various types of PVDs have been employed to accelerate the rate of primary consolidation. With vacuum application, an effective airtight membrane placed over the surface can prevent the air leak, and the applied suction head will propagate along the surface and down the drains. The thickness of the surcharge fill required otherwise may be reduced by several meters thereby saving time for stage embankment construction and embankment removal. Once the soil has increased in shear strength, the post-construction soil deformation will be considerably less, thereby eliminating any risk of instability of the overlying infrastructure.

The reduced pore pressure dissipation and settlement in the field when PVDs were installed too close implies that the smear effect around the drains was probably overlapping to each others. Also, anisotropic properties and creep nature of the soil may also have contributed to differences between the predicted and measured lateral deformation due to the major drawbacks of the Cam-clay model.

ACKNOWLEDGEMENTS

The Authors gratefully acknowledge the financial assistance from Australian Research Council. CHEMSTAB, Queensland Department of Main Roads. Authors wish to thank Dr. Thevaragavan Muttuvel for providing the experimental data for the current investigation. A number of other current and past doctoral students of the first Authors, namely, Mr. Somalingam Balachandran, Ms. Pushpachandra Ratnayake, Dr. I Wayan Redana, Dr. Chamari Bamunawita, Dr. Iyathurai Sathananthan, Dr. Rohan walker and Dr. Briony Rankine are also contributed to the contents of this keynote paper. More elaborate details of the contents discussed in this paper can be found in previous publications of the first author and his research students in Geotechnique, ASCE. Canadian Geotechnical Journals, ICE Proceedings since mid 1990's.

REFERENCES

ASTM standards, (2004). Identification and classification of dispersive clay soils by the pinhole test. *Section 4, Construction*, Vol. 04.08: 826-833.

Abulnaga, B (2002), *Slurry systems hand book*, McGraw- Hill Handbooks, p.2.11

Balasubramaniam, A.S, Phienwej, N, Lin, D.G. Karuzzaman, A. H. M. Uddin, K and Bergado, D. T. 1998. Chemical stabilization of Bangkok clay with cement, lime and flyash additives, *Proceedings of 13th Southeast Asian Geotechnical Conference*, Taipei, Taiwan, 253-2581

Barron, R.A. (1948). Consolidation of fine-grained soils by drain wells. *Transactions ASCE*, 113: 718-754.

Bergado, D.T., R. Manivannan and A.S. Balasubramaniam (1996). Proposed criteria for discharge capacity of prefabricated vertical drains. *Geotextiles and Geomembranes*, 14: 481-505.

Bo, M. W., Chu, J., Low, B. K., and Choa, V. (2003). *Soil improvement: prefabricated vertical drain techniques*, Thomson Learning, Singapore.

Chemstab.2003.Technical Manual. CHEMSTAB Consulting Pty Ltd, Horsley, NSW Australia.

Chew, S. H., Kamruzzaman, A. H. M. & Lee, F. H. 2004. Physicochemical and engineering behaviour of cement treated clays. J. Geotech. Geoenviron. Engng, 130(7): 69–706.

Wan, C.F. and Fell, R. (2004). Investigation of erosion rate of soils in embankment dams. *Journal of Geotechnical and Geo-environmental Engineering*, ASCE, 130 (4): 373-380.
Hansbo, S. (1981). Consolidation of fine-grained soils by prefabricated drains. *Proc. 10th International Conference Soil Mechanics and Foundation Engineering*, Stockholm, 677-682.
Hird, C.C., Pyrahm, I.C., and Russel, D. (1992). Finite element modeling of vertical drains beneath embankments on soft ground. *Geotechnique*, 42(3): 499-511.
Holtz, R.D., Jamiolkowski, M., Lancellotta, R. and Pedroni, S. (1991). *Prefabricated vertical drains: design and performance, CIRIA ground engineering report: ground improvement*. Butterworth-Heinemann Ltd, UK, 131 p.
Indraratna, B., and Redana, I. W. (1998). Laboratory determination of smear zone due to vertical drain installation. *Journal of Geotechnical Engineering*, ASCE, 125(1): 96-99.
Indraratna, B., Balasubramaniam, A.S, and Khan, M. J. (1995) Effect of fly ash with lime and cement on the behaviour of soft clays. *Quarterly Journal of Engineering Geology*, 28: 131-142.
Indraratna, B., Bamunawita, C., and Khabbaz, H. (2004). Numerical modelling of vacuum preloading and field applications. *Canadian Geotechnical Journal*, 41: 1098-1110.
Indraratna, B., Nutalaya, P. Kugamenthira, N. (1991). Stabilization of a dispersive soil by blending with flyash. *Quarterly Journal of Engineering Geology*, 24: 275-290.
Indraratna, B., Rujikiatkamjorn C., and Sathananthan, I. (2005). Radial consolidation of clay using compressibility indices and varying horizontal permeability. *Canadian Geotechnical Journal*, 42: 1330-1341.
Indraratna, B., Muttuvel, T, Khabaaz, H and Armstrong, B. 2008. Predicting the erosion rate of chemically treated soil using a process simulation apparatus for internal crack erosion, *Journal of Geotechnical and Geo-environmental Engineering*, ASCE, Vol. 134 (6): 837-844.
Karol, R.H. (2003). Chemical grouting and soil stabilization, 3rd Edition, *Marcel Dekker Inc.*, New York
Lohnes, R.A., and Coree, B.J. (2002). Determination and Evaluation of Alternative Methods for Managing and Controlling Highway-Related Dust, Report TR 449, *Iowa Highway Research Board, Iowa Department of Transportation*, 32 pp.
Pengelley, A.D., Boehm, W.D., Rector, E and Welsh, J.P. (1997). Engineering experiences with in-situ modification of collapsible and expansive soils. Unsaturated Soil Eng. Practice, ASCE special publication No.68: 277-298
Puppala, A.J and Hanchanloet, S. (1999). Evaluation of a new chemical treatment method on strength and resilient properties of a cohesive soils. Paper No.990389, *Transportation Research Board*
Qian, J.H., Zhao, W.B., Cheung, Y.K. and Lee, P.K.K. (1992). The theory and practice of vacuum preloading. *Computers and Geotechnics*, 13: 103-118.
Queensland Department of Main Roads. (1992). *Sunshine Motorway Stage 2 – Interim Report on the Performance of the Trial Embankment Area 2A* (Ch 28490-28640). Materials and Geotechnical Services Branch, July, R1802.
Rajasekaran, G., Murali, K and Srinivasaragahvan,S. (1997). Effect of chlorides and sulphates on lime treated marine clays, *Soils Foundations* 37:105–115.
Rankine, B., Indraratna, B., Sivakugan, N., Wijeyakulasuriya, V., and Rujikiatkamjorn C. (2008). Foundation behaviour below an embankment on soft soils, *Geotechnical Engineering*, ICE (Accepted).

Sathananthan, I. and Indraratna, B. (2006). Laboratory Evaluation of Smear Zone and Correlation between Permeability and Moisture Content. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 132(7): 942-945.
Sathananthan, I., Indraratna, B., and Rujikiatkamjorn C., (2008). The evaluation of smear zone extent surrounding mandrel driven vertical drains using the cavity expansion theory. *International Journal of Geomechanics*, ASCE. (In press).
Seah, T.H. (2006). Design and construction of ground improvement works at Suvarnabhumi Airport. Geotechnical Engineering, *Journal of Southeast Asian Geotechnical. Society*, 37, 171-188.
Shang, J. Q., Tang, M., and Miao, Z. (1998). Vacuum preloading consolidation of reclaimed land: a case study. *Canadian Geotechnical Journal*, 35: 740-749.
Sharma, J. S., and Xiao, D. (2000). Characterization of a smear zone around vertical drains by large-scale laboratory tests. *Canadian Geotechnical Journal*, 37(6): 1265–1271.
Sherard J.L., Dunnigan L.P., Decker R.S., Steele E.F. (1976). "Pinhole test for identifying dispersive soils." *Geotechnical special publication,* ASCE, No: 32: 280-296.
Thevaragavan, M. (2008), Erosion rate of chemically stabilized soils incorporating tensile stress-deformation behaviour, *Ph.D thesis submitted to University of Wollongon*g, Australia.
Tingle, J.S and Santori, R.L. (2003). Stablisation of clay soils with non traditional additives. Transporation research record No.1819, National Research Council, Washington. D.C: 72-84.
Uddin, K., Balasubramaniam, A.S. and Bergado, D. T. (1997). Engineering behaviour of cement treated Bangkok soft clay. *Geotechnical Engineering, SEAGS.* 28(1): 89-119
Walker, R. and Indraratna, B. (2007). Vertical drain consolidation with overlapping smear zones. *Geotechnique*, 57 (5), 463-467.
Yan, S.W. and Chu, J. (2003). Soil improvement for a road using a vacuum preloading method. *Ground Improvement*, 7(4), 165-172.